

Ambient response of a unique performance-based design tall building with dynamic response modification features

Mehmet Çelebi^{1*,†}, Moh Huang², Anthony Shakal², John Hooper³ and Ron Klemencic³

¹*Earthquake Science Center, USGS, Menlo Park, California, USA*

²*California Geological Survey, Sacramento, California, USA*

³*Magnusson Klemencic Associates, Seattle, Washington, USA*

SUMMARY

A 64-story, performance-based design building with reinforced concrete core shear walls and unique dynamic response modification features (tuned liquid sloshing dampers and buckling-restrained braces) has been instrumented with a monitoring array of 72 channels of accelerometers. The responses of the building to ambient motions from ground or wind were recorded and analyzed to identify modes and associated frequencies and damping. Not unexpectedly, the low-amplitude dynamic characteristics are considerably different than those computed from design analyses. Nonetheless, these computed values serve as a baseline against which to compare future strong shaking responses. Such studies help to improve our understanding of the effectiveness of the response modification features at various levels of shaking, to evaluate the predictive capabilities of the design analysis tools and to improve similar designs in the future. Copyright © 2013 The Authors. *The Structural Design of Tall and Special Buildings* published by John Wiley & Sons Ltd.

Received 07 February 2013; Revised 26 March 2013; Accepted 31 March 2013

KEY WORDS: tall building; performance base; buckling-restrained braces; sloshing damper; core shear wall; ambient response; mode shapes; spectra

1. INTRODUCTION

A new, landmark building decorates the panorama of San Francisco, California. Completed in 2008, the slender 64-story tall, reinforced concrete shear-wall core building (hereafter referred to as ‘the building’) is described as the tallest building in the USA designed using performance-based seismic design (PBSD) procedures (MKA, written communication, 2012). The unique structural dynamics modification features such as buckling-restrained braces (BRBs) and tuned liquid sloshing dampers (TSDs) qualify the building to be the tallest PBSD in the world, using BRBs (MKA, written communication, 2012). Also, One Rincon Hill Tower is the first building in California to have a liquid tuned mass damper.

The purpose of this paper was to introduce this unique building and its extensive seismic instrumentation recently deployed under a cooperative project between the California Strong Motion Instrumentation Program (CSMIP) of the California Geological Survey and the National Strong Motion Project (NSMP) under the Advanced National Seismic Systems managed by the United States Geological Survey (USGS). This instrumentation project (Station no. CSMIP 58389; NSMP 8389) includes a 72-channel seismic monitoring system that streams real-time acceleration data from multiple floors starting at level 1 (basement parking level P4) up to level 64 (roof). Detailed discussion on the choice of sensor locations throughout the building is outside the scope of this paper but can be found in the work of Huang *et al.* (2012) as well as Çelebi *et al.* (2012). General information on structural monitoring procedures and suggestions for deployments of accelerometers can be found in the work of COSMOS (2001) and Çelebi (2004).

*Correspondence to: Mehmet Çelebi, Earthquake Science Center, USGS, Menlo Park, CA 94025, USA.

†E-mail: celebi@usgs.gov

Recognizing that there is no known response data from such a unique design, we obtained, using on-demand recording capability of the data acquisition system, several sets of records of the building response to ambient motions to investigate the elastic behavior of the building at low amplitudes. As noted in previous studies of other structures (e.g. Çelebi, 2004; Çelebi *et al.*, 2013a), the response characteristics determined at low amplitude levels of shaking are generally significantly different from those expected at higher levels of shaking caused by earthquakes and strong winds. In particular, the response of this building during strong shaking is expected to be significantly altered by the special structural features described above. Thus, the dynamic characteristics identified from the response of the building to ambient motions will be the baseline elastic response. The scope of this paper includes descriptions of the building and the monitoring system, and a discussion of the building behavior as inferred from analyses of ambient response data. Finite element model (FEM) analyses were not performed as part of this study, but references are made to those performed by the designers. In this paper, we used spectral analyses techniques as described in Bendat and Piersol (1980) and coded in the public domain software, MATLAB (Mathworks, 2012). We also used system identification techniques also available in MATLAB (Mathworks, 2012) to extract mode shapes and associated frequencies and damping.

2. THE BUILDING

Figure 1 shows a Google Earth© 3-D street view of the building in proximity to another San Francisco landmark, the San Francisco Bay Bridge (SFBB). The tower, with 376 condominium homes, was designed in 2004, and the construction was completed in 2008. The west anchorage structure for suspension cables of the SFBB is approximately 100 m from the building and thus may be a significant source of ambient vibration for the building.

2.1. The structural system

In Figure 2, the main skeletal core shear wall and ‘outrigger’ BRB system of the building are shown. The total height of the building is 188.31 m (617.83 ft), which exceeds the 73.15-m (240 ft) limit specified in the code for a typical concrete shear-wall structure. The structural and architectural

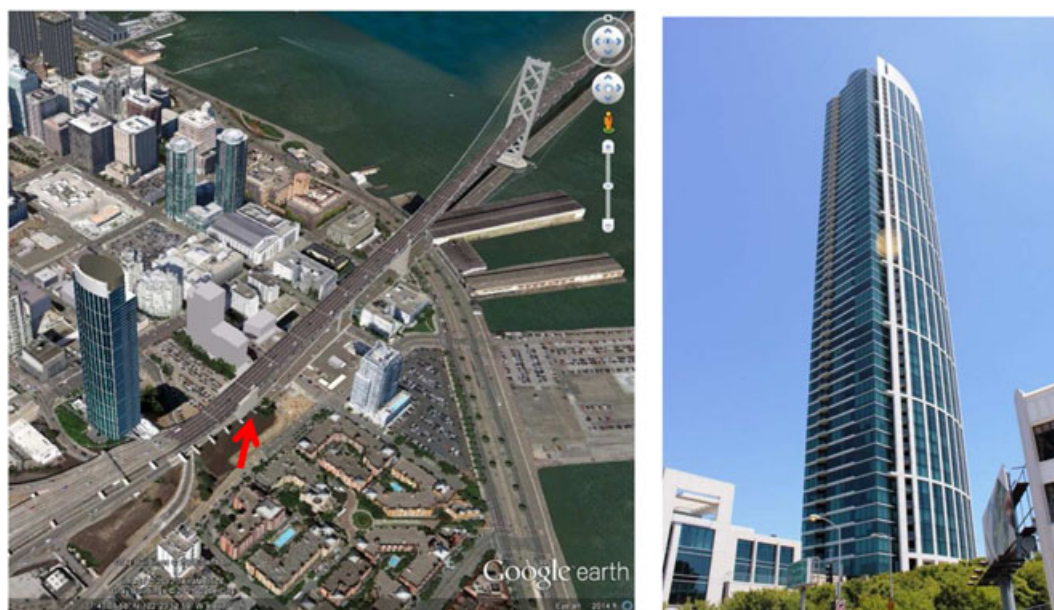


Figure 1. (Left) Google Earth© street view showing the building (left center) and vicinity, and the west end of the SFBB. Arrow points the west anchorage structure of the SFBB, which is approximately 100 m from the building. (Right) Close-up picture of the building.

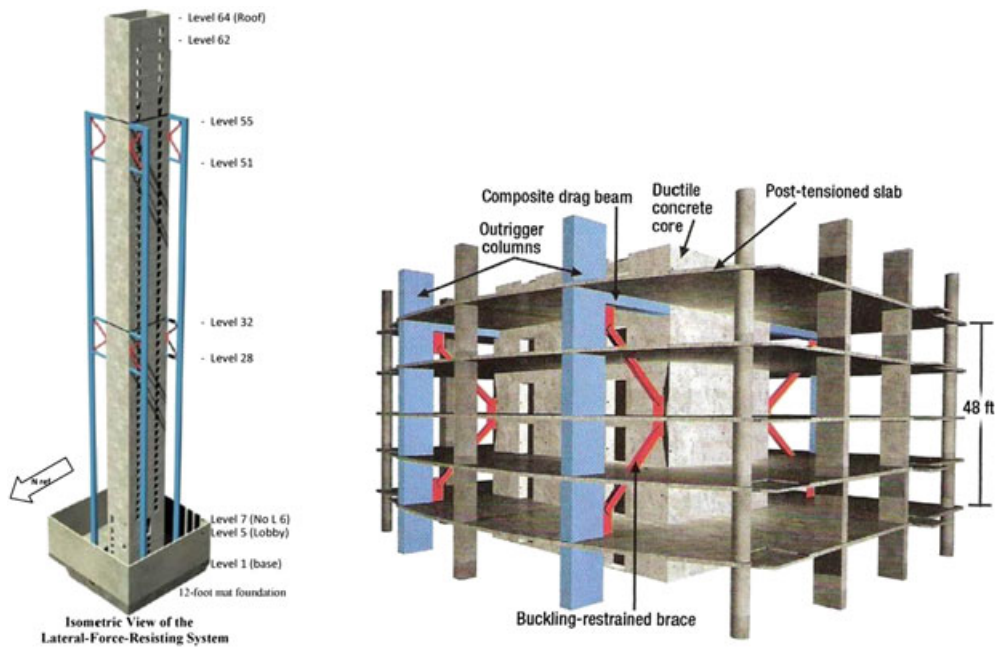


Figure 2. (Left) Core shear-wall lateral force resisting system of One Rincon Hill Tower with BRB system in the E-W direction and (right) the outrigger systems, the core shear wall, slabs and BRB details (from Klemencic *et al.*, 2006). Reference north is also shown.

systems of the building were designed according to the 2001 San Francisco Building Code (SFBC) and based on PBSO (Klemencic *et al.*, 2006; Klemencic, 2008). However, the design was also peer reviewed because of its uniqueness.

The vertical load carrying system of the building consists of concrete flat slabs supported by concrete columns and core shear walls. Typical residential floors are ~20.3-cm-thick (8 in.) post-tensioned slabs spanning between the center core and perimeter concrete columns. Post-tensioned tendons used in the concrete slabs are 1.27 cm (0.5 in.) in diameter (7-wire strand) with an ultimate tensile strength of 1862 MPa (270 ksi). The floor slab at the center core is typically 30.5 cm (12 in.) thick.

The lateral force resisting system of the building comprises a concrete ductile core wall system with added concrete outrigger columns in the transverse direction. An isometric view of the lateral force resisting system is shown in Figure 2. The core wall system is arranged in the form of perforated structural tube. The outrigger columns are connected to the core with steel BRBs at two locations and terminate at level 55. Lateral forces are carried by the floor diaphragms to the shear walls. Moments and shear forces are delivered to the foundation by the shear walls.

Figure 3 depicts several typical floor plan views displaying core shear walls, 'outrigger' columns, orientations and approximate locations of installed accelerometers and also the two TSD pools at the 62nd level. Tall outrigger columns integrate into core shear walls with the steel buckling resistant braces (BRBs). The water tanks (about 1.52 m (5 ft) tall) located between levels 62 and 63 (Post, 2008, 2012) are designed to act as a tuned mass damper in order to reduce the sway from seismic events or strong winds and to enhance human comfort from frequent wind storms with return periods of 1–10 years.

The floor plan is rectangular at the base level with a footprint of about 34.44 m by 41.76 m (113 ft by 137 ft). A curved outer facade is located on the west side between the seventh level and upper levels. Below the seventh level, concrete shear walls surround the building.

The plan views also show the true north and reference north (hereinafter referred to as NS). A typical floor area of this condominium building is approximately 880 m² (9500 ft²). The thickness of the core shear walls from level 1 (parking level 4) to the 32nd level is 81.3 cm (32 in.), from the 32nd level to the 55th level is 71.3 cm (28 in.) and from the 55th level to the top of the building is 61.0 cm (24 in.). Thus, with these shear walls, the wall-to-floor area percentages change from 2.4% to 3.9%, making the

building one with considerably higher wall-to-floor percentages and comparable with average percentages of the shear-wall buildings in Chile that performed well during the 1985 Valparaiso ($M=7.8$) and 2010 Maule ($M=8.8$) earthquakes (Çelebi *et al.*, 2013a, 2013b). Outrigger concrete column thicknesses generally follow those of the core shear wall and are generally 2.29 m (7.5 ft) wide. The maximum size of the outrigger columns is 81.3 cm by 228.6 cm (2 ft 8 in. by 7 ft 6 in.) at the base level. All reinforcement used in seismic resisting elements is in conformity with the ASTM A-706, Grade 60 standards. The minimum ultimate compressive strength of the concrete at 28 days is 34.47 MPa (5000 psi) for the basement walls and foundation walls. The concrete strength at 56 days is 37.92 MPa (5500 psi) for post-tensioned floor slabs and varying between 41.37 and 55.16 MPa (6000–8000 psi) for columns and shear walls. If the outrigger and other columns are considered, the lateral force resisting elements (columns and walls) to the floor area percentages increase by about 35%, providing the building a comparatively large stiffness and strength. In general, floor slabs are 30.5 cm (12 in.) throughout the core but changes to 20.3 cm (8 in.) outside of the core.

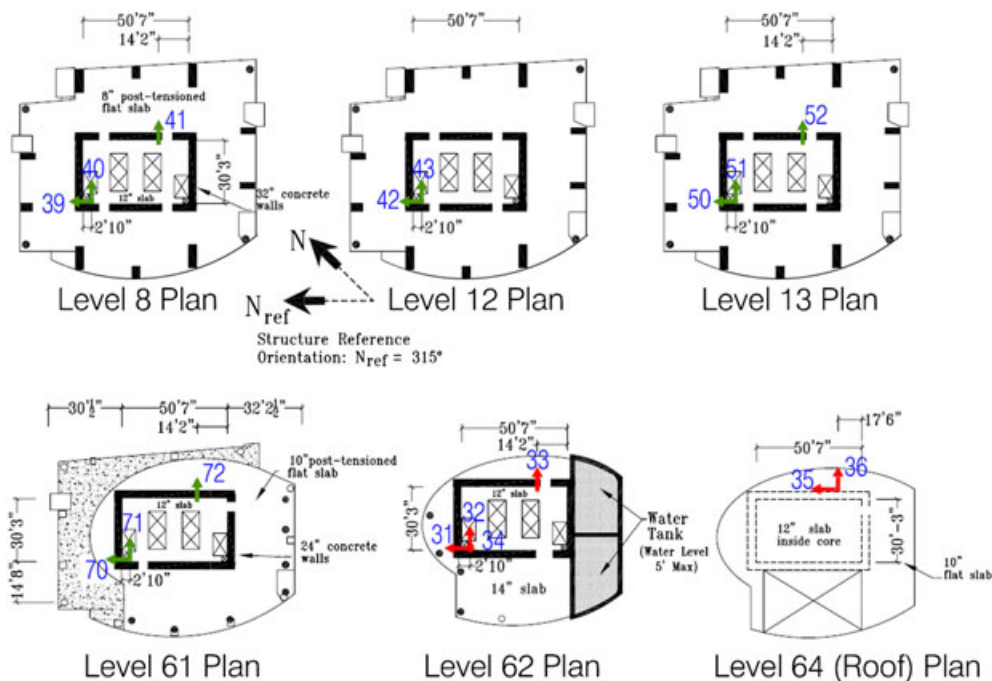


Figure 3. Typical plan views exhibiting sensor locations (green and red arrows), general dimensions and the core shear wall and outrigger columns (www.stongmotioncenter.org, last visited 29 July 2012). Note the building north reference direction (N_{ref}), which is termed NS in this paper.

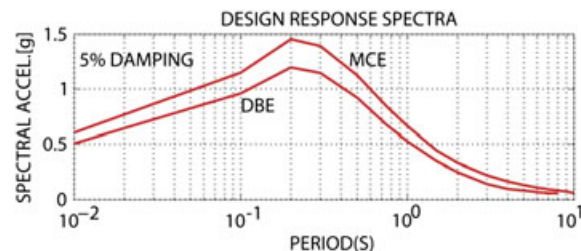


Figure 4. Design response spectra for DBE and MCE.

2.2. Underlying geology and foundation

The building has a 3.66 m (12 ft) mat foundation on Rincon Hill of San Francisco. The area has a geological formation described as a clastic sedimentary outcrop of sandstone and shale (Schlocker, 1974). According to the soil report (Treadwell and Rollo, Inc., 2004), the allowable bearing pressure is 1436 kPa (30 ksf). Foundation design and analysis were carried out using an FEM (Winkler-Foundation method (Das, 1984)). Demands on the mat foundation were determined through nonlinear analysis.

Fumal (1978, 1991) confirmed this description as sandstone and shale of Franciscan assemblage and provides an average 30-m shear wave velocity of $V_s = 745 \pm 140$ m/s for Rincon Hill. With assumption of 30-m depth for this average shear wave velocity, a site period of $T_s \sim 0.16$ s was computed. Although the actual site period possibly may be much shorter, in either case, it is much shorter than the fundamental period of a 64-story building (~ 5 – 6 s), even with the response modification features. Thus, site effects are not expected to be a significant factor in the responses of the building, at least for the fundamental mode. In the site report of design documents, the site is classified as B (Treadwell and Rollo, Inc., 2004).

2.3. Seismic design

According to the structural plans, key design parameters of the building are as follows:

- Live loads for typical floors were taken as 1.92 kPa (40 psf) and 1.20 kPa (25 psf) for the roof, whereas partition dead load was assumed to be 0.96 kPa (20 psf).

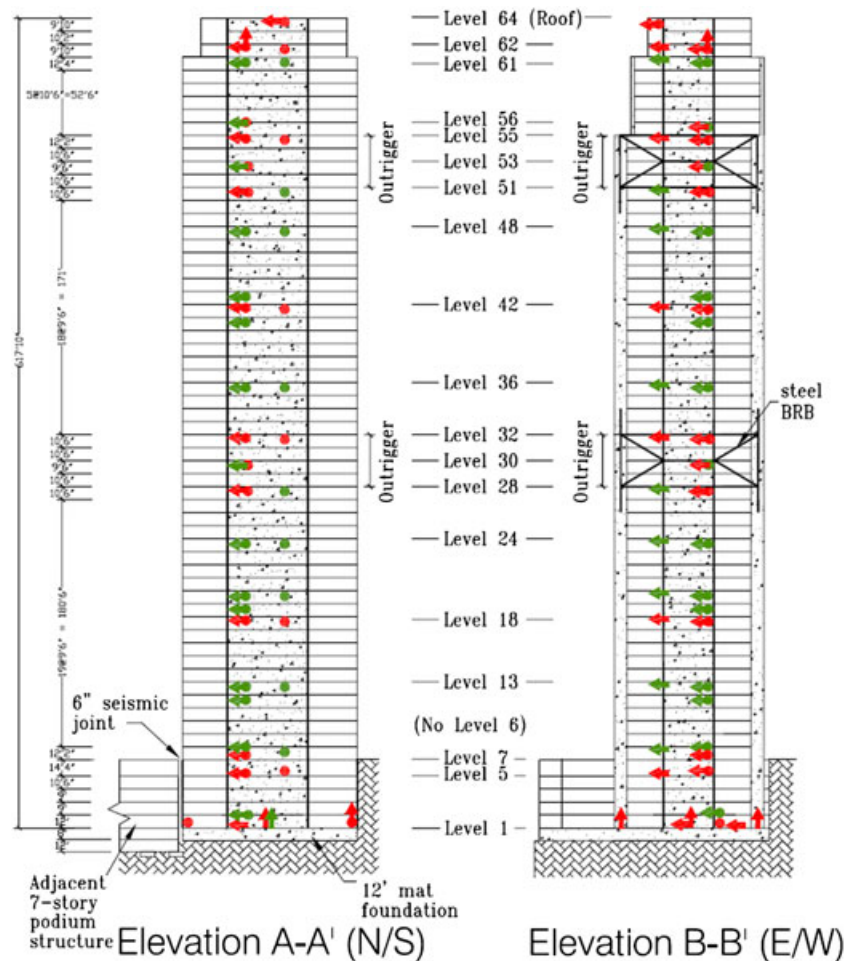


Figure 5. Vertical sections of the building showing locations of the accelerometers along the height of the building (www.stongmotioncenter.org, last visited 29 July 2012). Red and green colors refer to channels installed by the CSMIP and USGS NSMP, respectively.

- Equivalent static force analysis was conducted on the basis of a site class B soil condition with the following lateral load coefficients: zone 4; $I = 1.0$; $N_a = 1.0$; $N_v = 1.04$. The response reduction factor was taken as 4.5 for the structural system composed of shear wall/bearing wall.
- Wind loads were computed on the basis of wind tunnel testing conducted at the University of Western Ontario, Canada. Peak building wind acceleration was limited to 20 mg, considering a 10-year return period for the input wind forcing function.
- Site-specific response spectra were developed for both the design basis earthquake (DBE) and maximum considered earthquake (MCE) levels. The code-based elastic design was only performed for the DBE earthquake level.
- The time-history records were selected and scaled to be consistent with the site-specific MCE response spectrum. Seven pairs of ground motions were used in the nonlinear response analysis.

The design of the lateral force resisting system follows a two-step process. First is the elastic analysis and design considering the wind loads and the DBE level earthquake forces. The second stage is composed of nonlinear response analysis using strong-motion records scaled to the MCE ground shaking.

Both DBE and MCE levels of analyses were performed for 5% damped spectra corresponding to 10% probability of exceedance in 50 and 100 years, respectively. The two design response spectra are shown in Figure 4 (MKA, written communication, 2012). For DBE elastic analysis, an FEM model of the building was subjected to a DBE spectrum scaled to SFBC Section 1631.5. Corresponding zero-period accelerations for the DBE and MCE are 0.505 g and 0.602 g, respectively. Both response spectrum and nonlinear time-history analyses were performed by the designers. Ground motions recorded at select stations during seven large earthquakes with magnitudes ranging from 6.9 (1989 Loma Prieta, Los Gatos PC station) to 7.9 (2002 Denali, Pump Station 10) have been used for nonlinear analyses. For base shear evaluation at DBE level, SFBC period equation 30.8: $T = C_t (h_n)^{0.75}$ for $h_n = 577.33$ ft was computed as 2.36 s ($f = 0.42$ Hz). According to SFBC, this can be increased by 30% to $T = 3.06$ s ($F = 0.327$ Hz). These values are later compared with other characteristics identified in our analyses of the response data.

Table 1. Distribution and labeling of horizontal channels along the height of the building.

| Level | H (m) | H (ft) | Channel numbering (used in analyses) | | |
|-------|---------|----------|--------------------------------------|-----|-----|
| | | | NS | EW1 | EW2 |
| 1 | 0 | 0 | 37 | 38 | 6 |
| 5 | 12.34 | 40.5 | 7 | 8 | 9 |
| 7 | 16.71 | 54.83 | 10 | 11 | |
| 8 | 20.41 | 67 | 39 | 40 | 41 |
| 12 | 32 | 105 | 42 | 43 | |
| 13 | 34.9 | 114.5 | 50 | 51 | 52 |
| 18 | 49.38 | 162 | 12 | 13 | 14 |
| 19 | 52.27 | 171.5 | 44 | 45 | |
| 20 | 55.17 | 181 | 53 | 54 | 55 |
| 24 | 66.75 | 219.66 | 56 | 57 | 58 |
| 28 | 78.33 | 257 | 15 | 16 | 65 |
| 30 | 84.73 | 278 | 66 | 17 | |
| 32 | 91.13 | 299 | 18 | 19 | 20 |
| 36 | 103.72 | 337 | 59 | 60 | 61 |
| 41 | 117.7 | 384.5 | 46 | 47 | |
| 42 | 120.1 | 394 | 21 | 22 | 23 |
| 43 | 122.99 | 403.6 | 48 | 49 | |
| 48 | 137.46 | 451 | 62 | 63 | 64 |
| 51 | 146.46 | 480.5 | 24 | 25 | 67 |
| 53 | 152.55 | 500.5 | 68 | 26 | |
| 55 | 159.46 | 523.17 | 27 | 28 | 29 |
| 56 | 162.67 | 533.67 | 69 | 30 | |
| 61 | 179.22 | 588 | 70 | 71 | 72 |
| 62 | 185.21 | 607.83 | 31 | 32 | 33 |
| 64 | 188.31 | 617.83 | 35 | | 36 |

The building meets the code-specified drift limits for nonlinear analysis. The nonlinear seismic behavior of the structure is governed by coupling beam flexural behavior and flexural yielding of the wall near the ground level. Other potential mechanisms and actions are verified to remain elastic under the forces corresponding to the nonlinear time-history analysis. These actions include wall shear, wall flexure outside of the intended hinge zone, foundation and diaphragms (Klemencic *et al.*, 2006).

Table 2. Summary of ambient data recorded on-demand from the monitoring system.

| Date (YYYYMMDD) time (HHMM) | Length of data (s) | Raw data (samples per second) |
|-----------------------------|--------------------|-------------------------------|
| 201206011459 | 120 | 200 |
| 201206012159 | 120 | 200 |
| 201206040921 | 120 | 200 |
| 201207022012 | 240 | 200 |
| 201207030159 | 298 | 200 |

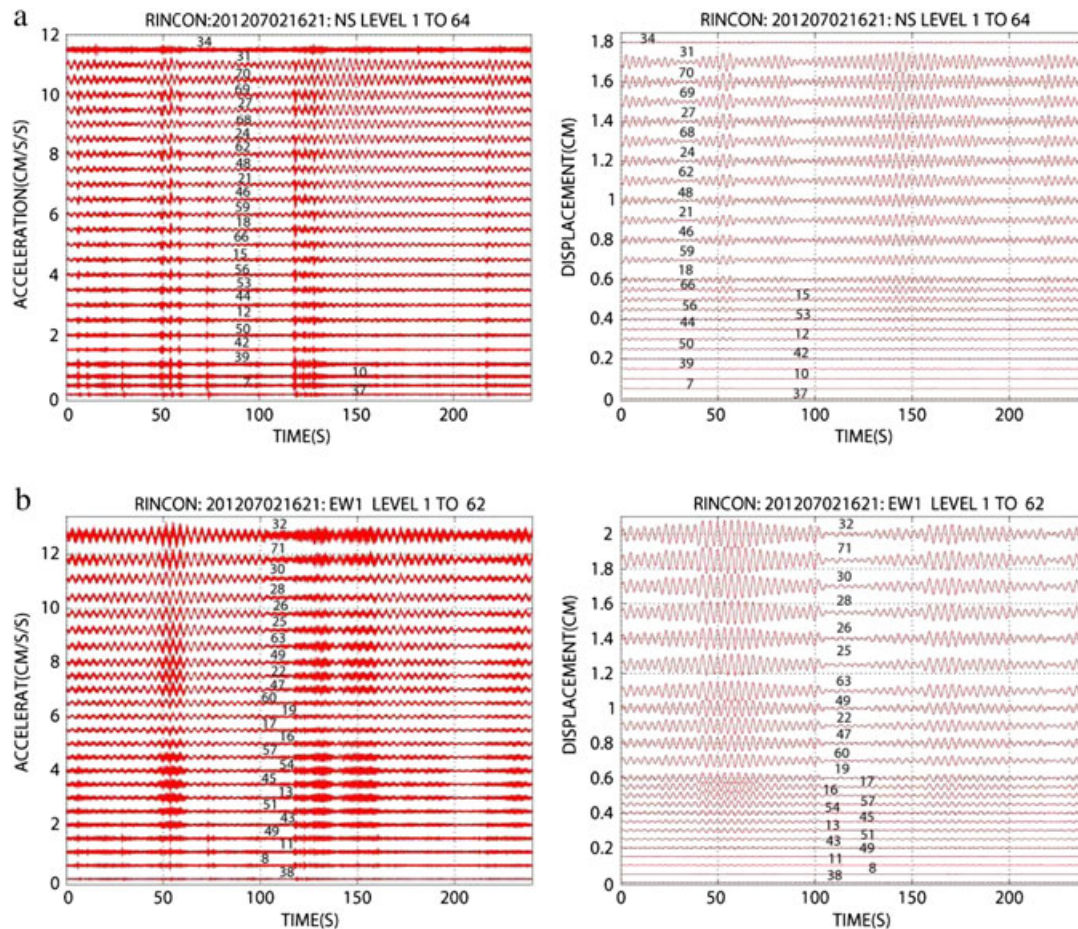


Figure 6. (a) Time-history plots of accelerations (left column) and displacements (right column) of translational (NS) motions for data set of 2 July 2012 (201207021621). (b) Time-history plots of accelerations (left column) and displacements (right column) of translational (EW1) motions for data set of 2 July 2012. (c) Time-history plots of accelerations (left column) and displacements (right column) of translational (EW2) motions for data set of 2 July 2012. (d) Time-history plots of relative torsional accelerations (left column) and relative torsional displacements (right column) of motions for data set of 2 July 2012.

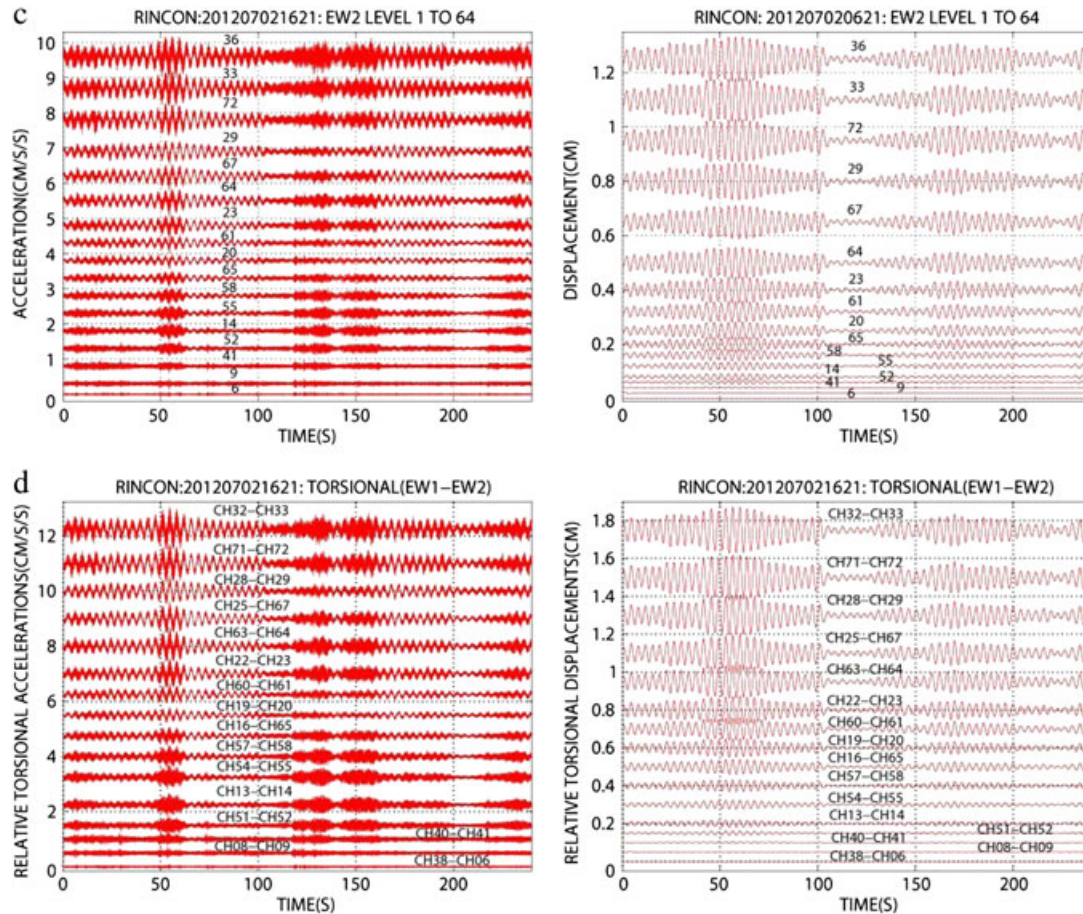


Figure 6. (Continued)

3. SEISMIC INSTRUMENTS AND SUMMARY OF AMBIENT DATA

The deployment of the 72 accelerometers (Figure 5) throughout the building is designed to capture its translational, torsional and vertical motions, and specifically to measure the motions at the levels where the outrigger BRBs and TSDs are located. Table 1 displays a cross-reference of the horizontal accelerometers at different levels. The four vertical channels (1–3 at level 1 and 34 at level 62) are not listed in the table because soil structure interaction (e.g. rocking) is not expected and related analyses are not performed due to the underlying geology and to the tall and slender design of the building.

The accelerometers used in the building are Kinemetrics EpiSensors¹ with ± 4 g full-scale recording capability, and the recorder system is a Kinemetrics Granite. The accelerometers are powered via cables from the recorder with ± 12 -V DC and have a power consumption of 1.6 A and ± 2.5 -V output range. The analog signals from the sensors are initially digitized at a very high sampling rate within the recorder, and then the digital data are multiplied by a calibration constant on the basis of the voltage output and decimated with application of an anti-alias filter to the desired sampling rate. USGS obtains and serves the digitized data at 200 samples per second.

Ambient data used in this paper were obtained on demand from the monitoring system of the building. Table 2 summarizes the recording intervals and length of the data. We used four of these data sets in this paper to infer the behavior of the building and repeatability of the results during

¹Mentioning commercial names in the manuscript is for information only and does not indicate endorsement of the manufacturer or the products.

low-amplitude ambient motions. The largest peak amplitude of acceleration among all the data sets is <1.5 gals. During processing of the data, the filters used are a cosine taper from 45 to 50 Hz and an acausal Butterworth with a corner of 0.17 Hz and $nroll=2$. USGS in-house BAP strong-motion data processing software was used to process the data (Converse and Brady, 1992).

4. DATA ANALYSES

4.1. Time-history plots of sample data set

Figure 6 shows time-history plots of accelerations and displacements for the data set obtained on 2 July 2012. In these plots, accelerometer channel organization described in Table 1 (translational NS, EW1 and EW2 alignments and torsional (EW1–EW2)) are used.

4.2. Spectral analyses

Detailed spectral analyses of only the data set 201206040921 are presented. Summary cross-spectrum (S_{xy}), phase angle and coherency plots for other data sets are also provided to exhibit repeatability in identification of translational and torsional modal frequencies.

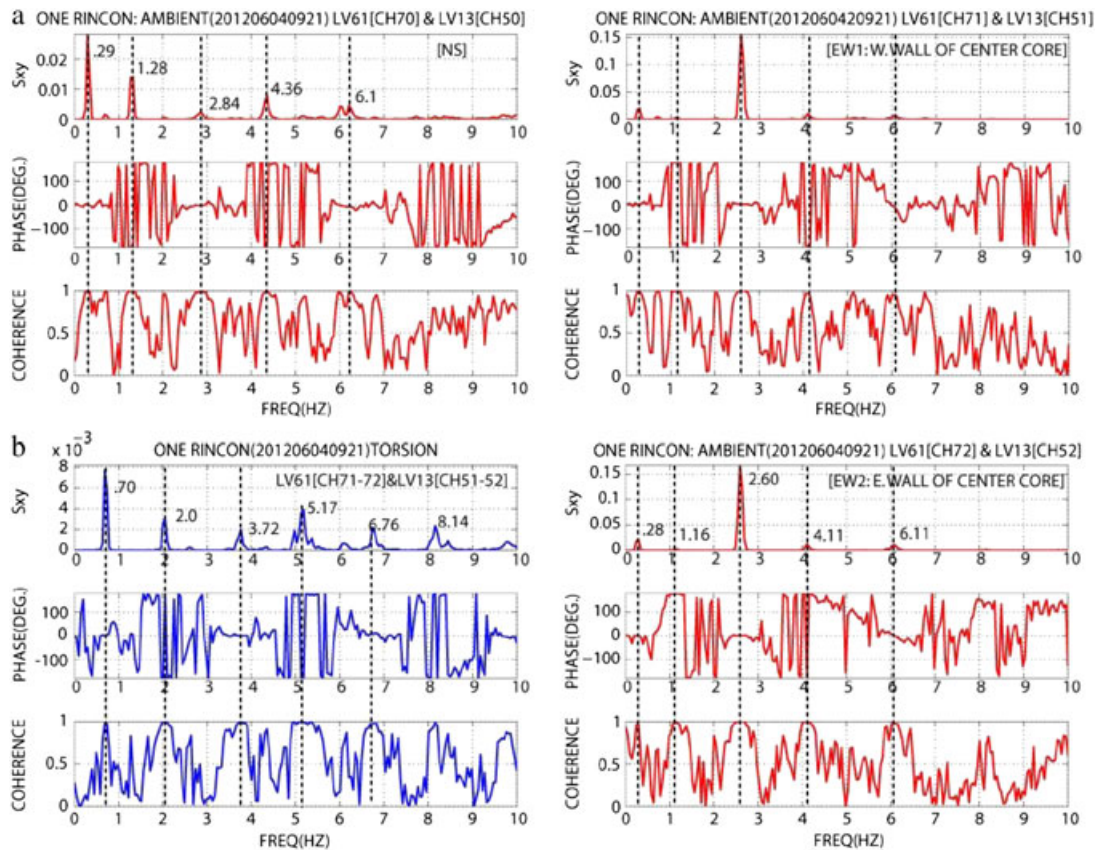


Figure 7. (a) Using data from levels 61 and 13, cross-spectra, phase angle and coherency plots identify modal frequencies for (left) NS direction and (right) EW1 direction using channels aligned with west end of core shear wall. (b) Using data from levels 61 and 13, cross-spectra, phase angle and coherency plots identify modal frequencies for (left) torsion using data from levels that have two EW parallel channels (EW1–EW2) and (right) EW2 direction: using channels aligned with east end of core shear wall.

Figure 7 shows at least five modes in perfect to near-perfect coherence and consistent phase angles corresponding to the first to fifth modal frequencies for the translational (NS, EW1 and EW2) and torsional modes.

Summary cross-spectra plots for four data sets are provided in Figure 8 and confirm that the identified frequencies are consistent for all four data sets.

4.3. System identification, extraction of modal shapes, frequencies and damping

System identification analysis was performed using the ambient data to identify and/or validate key frequencies and compare them with those determined by spectral analyses. In this study, we used measured data from the building as output (only) to estimate a state-spaced model with a predefined

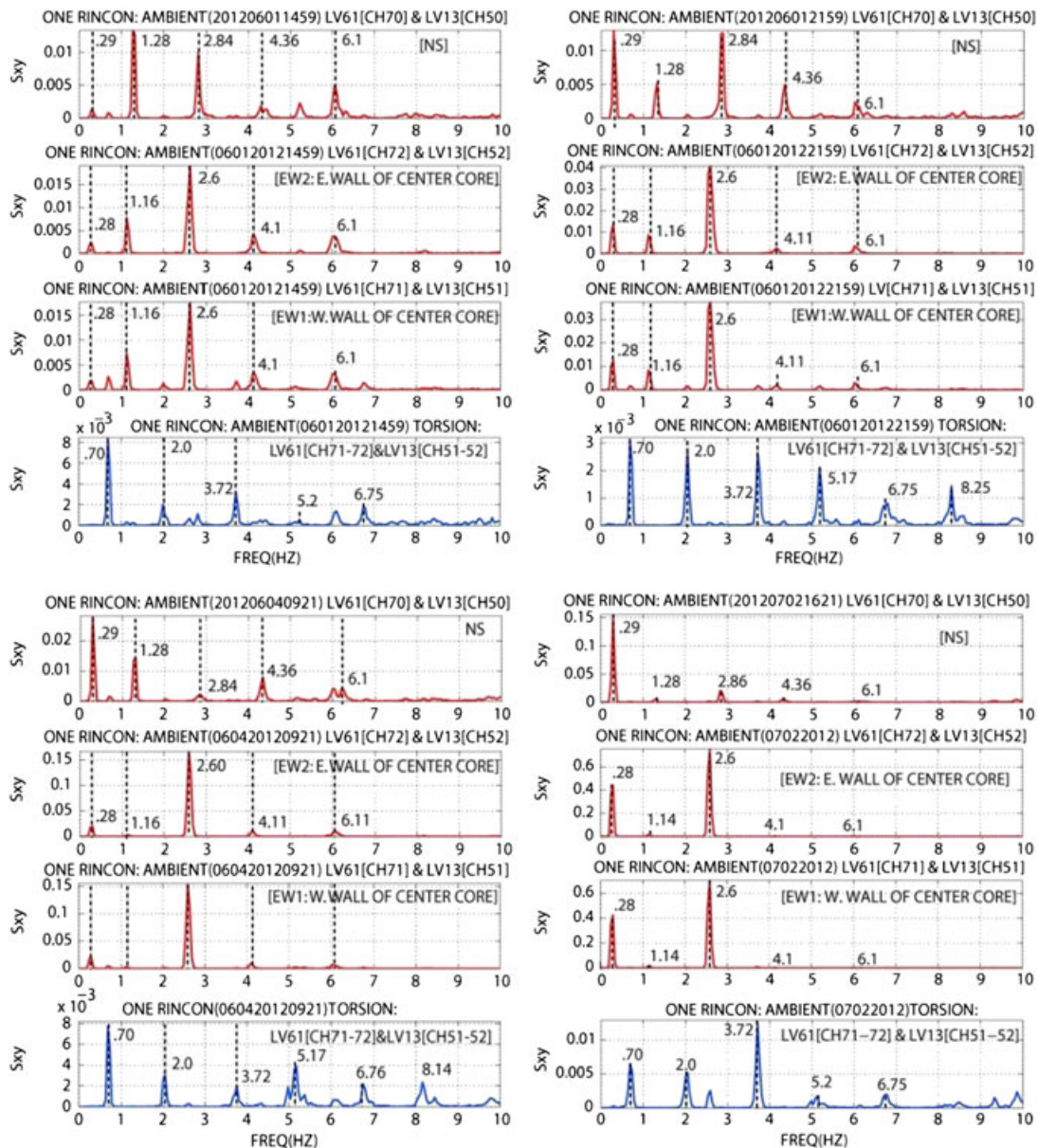


Figure 8. Summary cross-spectral amplitudes for translational (NS, EW1 and EW2) and torsional motions.

model order by using subspace method (N4Sid) as coded within MATLAB (Mathworks, 2012). Further details on the background of this method are not repeated here because they are provided in many publications including Mathworks (2012), Ljung (1987), Van Overschee and De Moor (1996) and Juang (1994).

Using only the 201206040921 data set, five mode shapes and corresponding frequencies and damping ratios for each of the output directions identified as NS, EW1, EW2 and EW1–EW2 (torsion) listed in Table 1 were computed and are presented in Figure 9.

As shown in this figure, the mode shapes conform reasonably to expectations but with some irregularities at higher modes. The mode shapes do not indicate any alterations due to BRBs or TSDs, which would not likely occur at these low-amplitude levels of ambient response. It is estimated that, at higher shaking levels, altered mode shapes due to the presence of BRBs and TSD would most likely display observable departure from those shown here, particularly at levels where the BRBs are attached to the main core.

4.4. Compilation and comparison of dynamic characteristics

In Table 3, dynamic characteristics identified from ambient data are compared with those computed during design analyses.

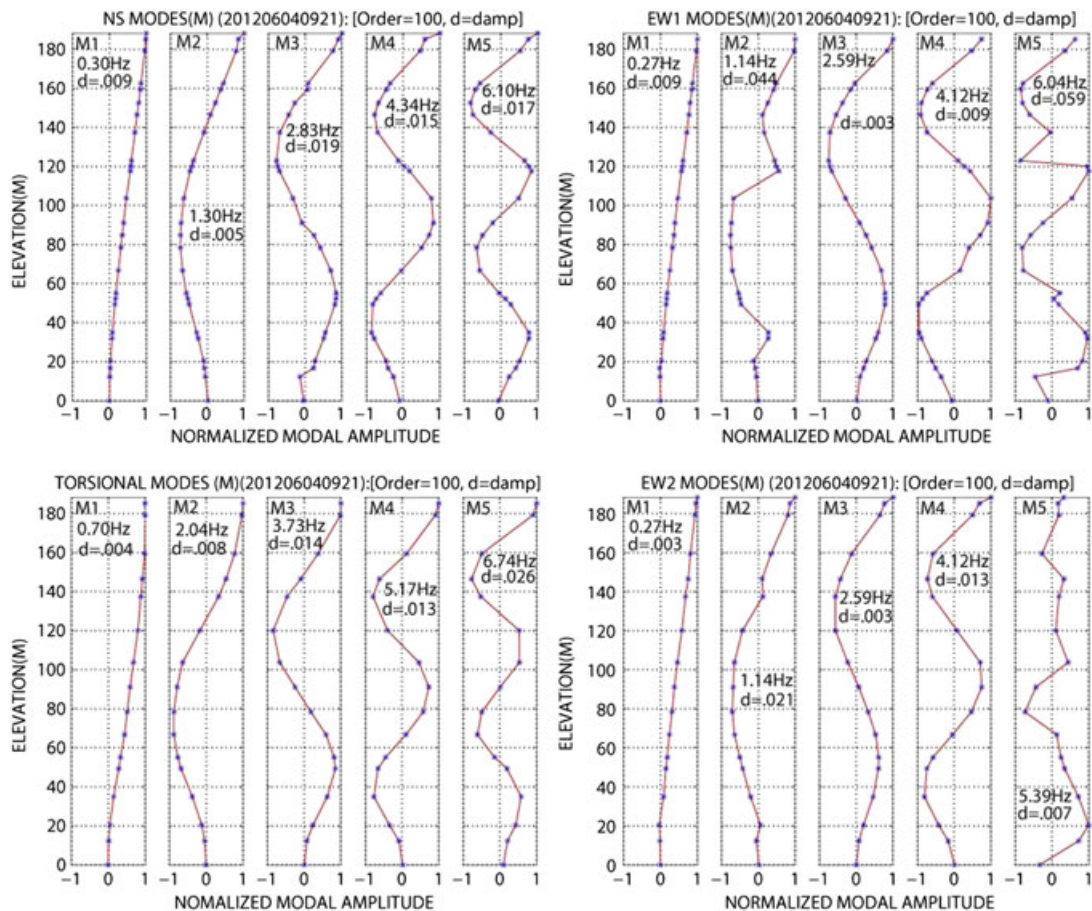


Figure 9. Modal shapes, frequencies and damping percentages for NS, EW1, EW2 and torsion (EW1–EW2) identified using data set 201206040921.

Table 3. Comparison of dynamic characteristics of the building.

| Code (SFBC Section 1630.2 equation 30.8) (MKA, written communication, 2012) | | | | | | |
|---|--------------------------------|--------------------------------|--------------------------------|-----------------------|--------------------------------|---------------------|
| $T = C_t(h_n)^{0.75} = 2.36 \text{ s } (f = 0.42 \text{ Hz})$ (increased 30% to 3.06 s or 0.327 Hz) | | | | | | |
| Mode | NS | EW | Torsion | | | |
| DBE level elastic analysis (MKA, written communication, 2012) | | | | | | |
| 1 | 4.91 s | 5.51 s | 2.26 s | | | |
| | 0.204 Hz | 0.182 Hz | 0.443 Hz | | | |
| MCE level nonlinear analyses procedures (MKA, written communication, 2012) | | | | | | |
| 1 | 4.53 s | 5.04 s | 1.86 s | | | |
| | 0.221 Hz | 0.198 Hz | 0.538 Hz | | | |
| Ambient data: spectral analyses (this study) | | | | | | |
| Mode | NS | EW | Torsion | | | |
| | $T \text{ (s)}/f \text{ (Hz)}$ | $T \text{ (s)}/f \text{ (Hz)}$ | $T \text{ (s)}/f \text{ (Hz)}$ | | | |
| 1 | 3.45 | 3.57 | 1.43 | | | |
| | 0.290 | 0.280 | 0.700 | | | |
| 2 | 0.781 | 0.862 | 0.5 | | | |
| | 1.28 | 1.16 | 2.0 | | | |
| 3 | 0.352 | 0.385 | 0.269 | | | |
| | 2.84 | 2.60 | 3.72 | | | |
| 4 | 0.229 | 0.244 | 0.192 | | | |
| | 4.36 | 4.10 | 5.2 | | | |
| 5 | 0.164 | 0.164 | 0.148 | | | |
| | 6.10 | 6.10 | 6.75 | | | |
| Ambient data: system identification (this study) | | | | | | |
| Mode | NS | | EW ^a | | Torsion | |
| | $T \text{ (s)}/f \text{ (Hz)}$ | $\zeta \text{ (%)}$ | $T \text{ (s)}/f \text{ (Hz)}$ | $\zeta \text{ (%)}$ | $T \text{ (s)}/f \text{ (Hz)}$ | $\zeta \text{ (%)}$ |
| 1 | 3.33 | 0.9 | 3.70 | 0.3–0.9 ^a | 1.43 | 0.4 |
| | 0.300 | | 0.270 | | 0.700 | |
| 2 | 0.769 | 0.5 | 0.877 | 2.1–4.4 ^a | 0.490 | 0.8 |
| | 1.30 | | 1.14 | | 2.04 | |
| 3 | 0.353 | 1.9 | 0.386 | 0.3 | 0.268 | 1.4 |
| | 2.83 | | 2.59 | | 3.73 | |
| 4 | 0.230 | 1.5 | 0.243 | 0.9–1.3 ^a | 0.193 | 1.3 |
| | 4.34 | | 4.12 | | 5.17 | |
| 5 | 0.164 | 1.7 | 0.166–0.186 ^a | 0.59–0.7 ^a | 0.148 | 2.6 |
| | 6.10 | | 5.39–6.04 | | 6.74 | |

^aVariations in EW direction is due to two analyses on EW1 and EW2 line-up of data.

5. DISCUSSION AND CONCLUSIONS

An extensive, state-of-the-art monitoring system was recently installed in a new core shear-wall reinforced concrete building having unique features (a combination of BRBs and TSDs) for modifying structural response during strong shaking. The building has not yet experienced strong shaking, but in order to serve as a baseline for future data analyses, be they of low amplitude or strong shaking, several records of the response of the building to ambient motions were acquired using a remote, on-demand feature of the monitoring system. For these low-amplitude shaking (maximum accelerations 1.5 gals), dynamic characteristics (periods, mode shapes and damping percentages) were identified using well-known spectral analyses and system identification techniques; these parameters are summarized in Table 3.

The NS, EW and torsional periods (frequencies) for the first five modes obtained by spectral analyses and system identification techniques are similar. The first modal damping percentages (<1%) are low, as can be expected from ambient motions (e.g. Çelebi, 2004; Çelebi *et al.*, 2013a). The mode

shapes of the building appear to be normal, which suggests that at these low-amplitude levels of shaking, the aforementioned BRB and TSD features contributed very little if any to the building behavior.

As might be expected, the ambient fundamental periods (frequencies) are considerably different from those computed using the code formula and from DBE level elastic and MCE level analyses. This may be explained by the fact that for the DBE and MCE level analyses, BRB and TSD characteristics were considered because they were expected to actively contribute to the building response at these higher levels of shaking. Future strong shaking data to be retrieved from the state-of-the-art seismic monitoring array of this building will be very valuable to assess its behavior, performance and effectiveness of the dynamic response modification features integrated into the building and facilitate comparison with the response characteristics identified from ambient data presented herein.

ACKNOWLEDGEMENTS

Data used in this study were obtained on demand by CSMIP from the seismic monitoring array of the building. The array was designed by CSMIP and NSMP staff and was deployed in a collaborative effort between CSMIP and USGS. Thanks are due to Chris Stephens, Manager of NSMP Strong Motion Data Center (USGS), for diligently processing some of the data. The homeowners' association of the building and the design engineering company, MKA, provided requisite information to carry out this study. Thanks are due to Roger Borchardt, Chris Stephens and Erdal Safak for reviews of the paper.

REFERENCES

- Bendat JS, Piersol AG. 1980. *Engineering Applications of Correlation and Spectral Analysis*. John Wiley and Sons: New York, N.Y.
- Çelebi M. 2004. Structural Monitoring Arrays – Past, Present and Future, PROC. NATO SFP Workshop on Future Directions on Strong Motion and Engineering Seismology, Kusadasi, Izmir, Turkey, May 17–21, 2004. *in* Future Directions in Strong Motion Instrumentation (eds. P. Gulkan and J. Anderson), NATO Science Series: IV. Earth and Environmental Sciences, Kluwer Academic Publishers, v. 58, pp. 157–179.
- Çelebi M., Huang M., Shakal S, Hooper J. and Klemencic. 2012. Ambient response of a unique performance-based design building with dynamic response modification features, *invited paper*, Proc. SMIP2012 Workshop, Sacramento, Ca. October 2, 2012.
- Çelebi M, Toksöz N, Büyüköztürk O. 2013a. Rocking behavior of an instrumented unique building on the MIT campus identified from ambient shaking data, *paper in print*, Earthquake Spectra.
- Çelebi M, Sereci M, Boroschek R, Carreno R, Bonelli P. 2013b. Identifying the dynamic characteristics of a dual core-wall and frame building in Chile using aftershocks of the 27 February 2010 (Mw = 8.8) Maule (Chile) earthquake, *paper in print*, Earthquake Spectra.
- Converse A, Brady G. 1992. BAP: Basic Strong-Motion Accelerogram Processing Software; Version 1.0 (USGS Open-File Report 92-296A).
- COSMOS. 2001. Proceedings of Invited Workshop on Strong-Motion Instrumentation of Buildings, CP-2001/04.
- Das B. 1984. *Principles of Foundation Engineering*. Brooks/Cole of Wadsworth Inc. Monterey, CA.
- Fumal T. 1978. Correlations between seismic wave velocities and physical properties of near-surface geologic materials in the southern San Francisco Bay region, California, USGS Open-File Report 78-1067.
- Fumal T. 1991. A Compilation of the Geology and Measured and Estimated Shear-Wave Velocity Profiles at Strong-Motion Stations that Recorded the Loma Prieta, California, Earthquake, USGS Open-File Report 91-311.
- Huang M, Shakal A, Petersen C, Çelebi M, Hooper J, Klemencic R. 2012. Strong Motion Instrumentation of a 62-story concrete core residential tower in San Francisco, Proc. SMIP12 Seminar on Utilization of Strong-Motion Data, Sacramento CA. Oct. 2, 2012.
- Juang JN. 1994. *Applied System Identification*, Prentice Hall: Upper Saddle River, NJ. 394 pages.
- Klemencic R. 2008. Performance Based Seismic Design – Rising, in STRUCTURE magazine, June 2008, p. 10–13.
- Klemencic R, Fry JA, Hooper JD. 2006. Performance-based design of tall reinforced concrete ductile core wall systems, *Annual Meeting of the Los Angeles Tall Building Structural Design Council – Alternative Procedures for Design of Tall Buildings*. p. 122–32.
- Ljung L. 1987. *System Identification: Theory and User*. Prentice hall: Englewood Cliffs, NJ.
- Mathworks. 2012. *Matlab Users Guide. System Identification Toolbox for use with Matlab*. The Mathworks Inc.: South Natick, MA.
- Post N. 2008. San Francisco's 590-ft Skyscraper Lifts Seismic Design Stature, in Engineering News Record, January 21, 2008, p. 22–26.
- Post N. 2012. Program Expected to Validate Extreme Seismic Engineering, in Engineering News Record, June 11, 2012, p. 16.
- Schlocker J. 1974. Geology of the San Francisco North quadrangle, California: U.S. Geological Survey Professional Paper 782, 109 p.
- Treadwell & Rollo, Inc. 2004. Updated Geotechnical Investigation, One Rincon Hill, 425 First Street, San Francisco, California, dated February 20, 2004.
- Van Overschee P, De Moor B. 1996. *Subspace Identification for Linear Systems*. Kluwer Academic Publishers: Dordrecht, The Netherlands.

AUTHORS' BIOGRAPHIES

Mehmet Çelebi received his BS degree in civil engineering from Middle East Technical University (METU), Ankara, Turkey; MSc from Stanford University, Stanford, California; and PhD from McGill University, Montreal, Canada. He has teaching, research and industrial experience. He served as professor at METU and San Francisco State University, Research Engineer at UC-Berkeley and Engineering Specialist in Bechtel Power Corporation in San Francisco, CA. He is currently employed as Senior Research Engineer at United States Geological Survey in Menlo Park, California. His research interests include monitoring of structures, analyses of response data from structures, site response and assessment of existing structures.

Moh Huang received his BS degree in civil engineering from National Taiwan University, Taipei, Taiwan, and MSc and PhD degrees in civil engineering from California Institute of Technology, Pasadena, California. He is currently employed at the California Geological Survey in Sacramento, California. His research interests include strong-motion instrumentation of structures, interpretation of structural response data, and effects of isolation bearings and energy dissipation devices on seismic response of buildings.

Anthony Shakal received his BS and MS degrees in engineering from the University of Wisconsin Milwaukee and his PhD in seismology from the Massachusetts Institute of Technology. He is currently employed as supervising manager of the California Strong Motion Instrumentation Program in the California Geological Survey in Sacramento, California. His current focus is on expanding and improving the measurement of strong motion for engineering analysis.

John Hooper received his BS in civil engineering from Seattle University and his MS in civil engineering from the University of California, Berkeley. He is currently the Director of Earthquake Engineering at Magnusson Klemencic Associates, a 180-person structural/civil engineering firm headquartered in Seattle, Washington. His current research interests include the next generation of performance-based design methodologies and advancing code-based seismic design approaches.

Ron Klemencic received his BS in civil engineering from Purdue University and his MS in structural engineering from the University of California, Berkeley. He is currently Chairman and CEO of Magnusson Klemencic Associates, a 180-person structural/civil engineering firm headquartered in Seattle, Washington. He is also a former Chairman of Council of Tall Buildings and Urban Habitat (CTBUH), serving from 2001 to 2006. Ron's current research interests include advances in structural and high-rise designs for seismically active regions.